SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING
AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

SEISMIC ASSESSMENT OF A "PLACA" BUILDING IN LISBON

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ABSTRACT

Stone masonry buildings represent the majority of the existing building stock in historic city centers throughout Europe. Due to their widespread availability in Lisbon area, "Placa" building is ones of the most typical examples of traditional Portuguese building types. They were built between 1930 and 1960, preceding the modern RC buildings and coming after the end of the period of "Gaioleiro" buildings (built between the late nineteenth and early twentieth century). Due to the fact that these buildings are still used for housing, improvement of the seismic resistance and increase of their safety is really important. In this paper, a brief description of the "Placa" buildings is presented and the results of the seismic assessment of one type of this existing building (the so-called "Rabo de Bacalhau") are discussed. The building was modelled by the equivalent frame method based on the solutions adopted in Tremuri software and in-plane capacity curves were defined with incremental static (pushover) analyses. Moreover, the damage distribution for the ultimate displacement and the seismic performance of the building in terms of maximum values displacement and maximum acceleration are also presented.

INTRODUCTION

Rehabilitation and strengthening of old masonry buildings represent one of the most important tasks for many cities located in regions of intense seismic activity, including Lisbon. To that respect, a complete identification and inspection surveys of old masonry buildings need to be performed. However, the exploratory studies of this kind are still very scarce in the case of traditional Portuguese buildings, especially for "Placa" buildings.

Focusing on traditional Portuguese masonry buildings, the outcomes of research preformed in the current state-of-the-art can only be partially applied when assessing and describing the seismic behavior of "Placa" buildings. This is mainly due to the fact that in "Placa" buildings the reinforced concrete is introduced for the first time in Portuguese masonry construction, thus resulting in a specific type of mixed buildings with masonry and reinforced concrete. However, the gradual inclusion of reinforced concrete slabs and frames also influences the overall seismic behavior; therefore "Placa" buildings are particularly vulnerable, to both, in plane (global) and out of plane behavior.

In the literature, there are only few scientific studies (Monteiro and Bento (2013), Lamego et. al, (2012), Lamego (2014)] that specifically target "Placa" buildings.

Since "Placa" buildings represent mixed masonry - reinforced concrete structures (RC), there are some studies in the literature (Cattari and Lagomarsino, 2013) that target similar type of masonry, but for structures that greatly differ from these Portuguese buildings. Moreover, there are also few references

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to experimental campaigns Tomaževič et al., (1990), Jurukovski et al., (1992), Tondelli et al., (2013) and numerical studies Augenti and Parisi (2009).
Due to this and the fact that these buildings still represent an important part of the Lisbon building stock, developments of the studies aimed to assess their seismic behavior and afterwards to propose strengthen solutions are crucial.
The main goal of the work herein presented is to analyze the seismic vulnerability of one type of "Placa" building, namely "Rabo de Bacalhau". It was assumed that the connections between walls and floors and walls were strengthened and thus the out-of-plane behavior prevented. Therefore, only the global seismic behavior of the building is assessed taken into account the in-plane contribution of the walls. The modeling of building structure was implemented by means of the equivalent frame method based on the solutions adopted in Tremuri software (Lagomarsino et al., 2013). Each masonry wall was discretized by a set of panels (piers and spandrels), in which the non-linear response was concentrated, connected by a rigid area (nodes). The masonry panels, as well as the reinforced concrete (RC) beams and columns, present on the 'Placa' building, were modelled as non-linear beams. Floor elements were modelled as orthotropic membrane finite elements. The structures capacity curves were determined by incremental nonlinear static (pushover) analysis.
Additionally, the damage distribution for the ultimate value of displacement is presented as well as the values of the target displacement and the maximum values of admissible ground acceleration for the two types of seismic action that should be considered in Lisbon (according to EC8 - CEN 2004 proposal) .

CHARACTERIZATION OF "PLACA" BUILDINGS

"Placa" buildings characterize the urban expansion of Lisbon on the decade between 30 and 60 of the twentieth century. This period began after the end of the period of "Gaioleiro" buildings and it is characterized as the last type of construction in Lisbon, using the masonry as a structural element.
Thus, this new type of construction corresponds with the transition period between the masonry and RC buildings in different areas of Lisbon, namely "Bairro dos Actores" and "Bairro de Alvalade" (Fig.1). These buildings are generally mid-rise structures with four to six stories (Fig. 2).

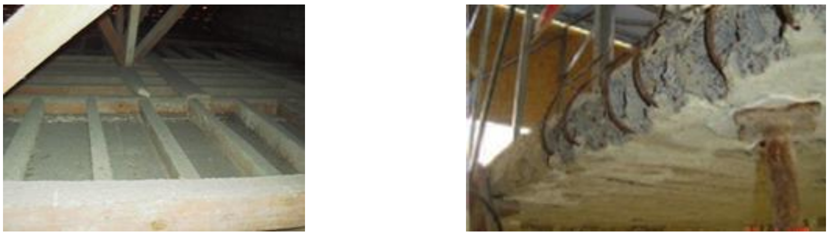
Figure 1 Location of "Placa" buildings in Lisbon

Figure 2“Placa” buildings in Lisbon

Foundation for this type of buildings was made by very stiff stone masonry and with a hydraulic mortar. Furthermore, in this period, the first reinforced concrete foundations appear in some buildings (RGCU, 1930). The façade walls are made of rubble stone masonry (with thickness between 0.4 m and 0.7m) with hydraulic or cement mortar (Fig. 3(a)). In some cases, these walls are characterized by decreased of thickness along the height of the building. The side (gable) walls are made with the same materials as façade walls (with reduced thickness) or with concrete blocks (Fig. 3(b)). The interior walls are built with brick masonry or concrete blocks, where the thickness can decrease in the height of the building. The partition walls are made with hollow brick masonry. As already mentioned, in these buildings the first RC elements started to appear. First, by means of RC peripheral lintels used to strengthen the timber floors and RC frames at the ground floor when larger spans and open spaces were needed. The timber floors (Fig. 4(a)) were after replaced by RC slabs (Fig. 4(b)): first only on the back balconies and the services room of the house (kitchen and bathrooms), whereas after this solution was extended to the whole floor, supporting the name ‘Placa’ (meaning RC slab) given to this typology of buildings. Generally, the concrete slabs have approximately 0.07m to 0.1m thicknesses. However, in some buildings the floors are still made of wooden beams placed perpendicular to the front façade walls.

(a) (b)

Figure 3 Examples of different walls used: a) façade wall with rubble masonry (Alegre, 1999), b) gable wall with concrete blocks (Mauro and Bento, 2012)



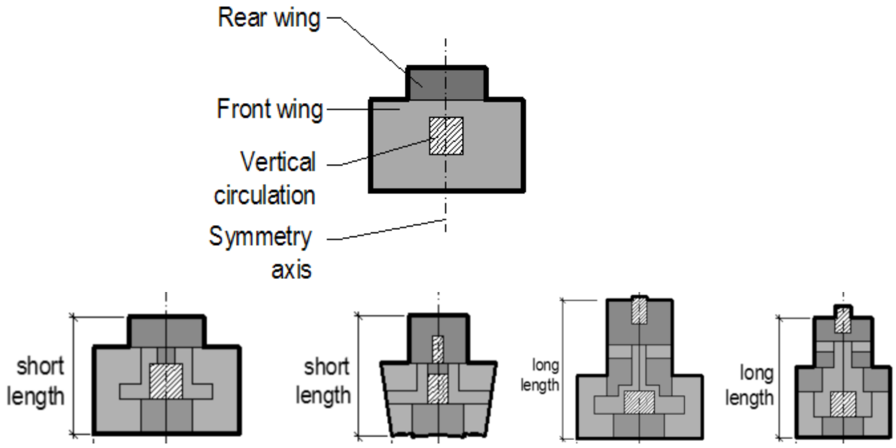
(a) (b)

Figure 4 Examples of floors (Mauro and Bento, 2012): (a) Timber floor on the roof, (b) Concrete slab

These buildings have two types of construction in the plan: i) rectangular (predominant type in the Bairro de Alvalade) and ii) “Rabo de Bacalhau” (predominant type in the “Bairro dos Actores”). The latter are characterized by buildings with a salient shape on the back of the building which is made of

RC frames and slabs; it can be divided in four sub-types: type A, type B, type C, type D (Eloy, 2012), as is shown in Fig. 5. These four types can be recognized by the following differences:

- Width of front wing;
- Depth of rear wing;
- Functional use of rear wing (only services area in the shallow rear wing and services, private and/or social areas in deep rear wing);
- Type and number of vertical accesses in building;
- Circulation typology within the dwelling;
- Number and type of rooms.

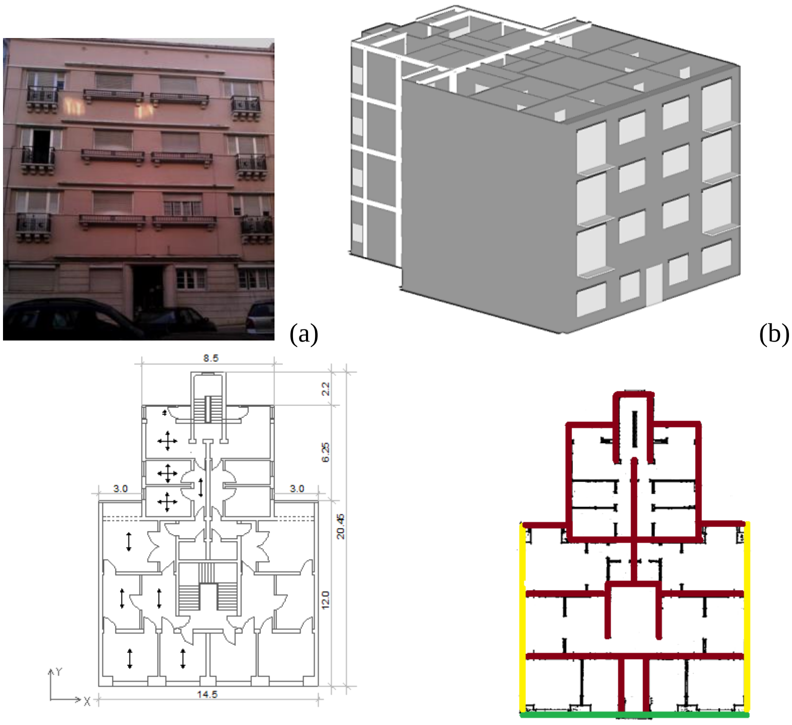


(a) (b) (c) (d)

Figure 5 Four types of "Rabo de Bacalhau dwellings": (a) Type A; (b) Type B; (c) Type C; (d) Type D (Eloy, 2012)

In this study, the selected building has a type D 'Rabo de Bacalhau' plan shape with 14.5 m x 20.5 m and four stories high with a total height of 12.3 m, as can be seen in Fig. 6. For the upper floors, the height of the stories is constant with 3.0 m, whereas the ground floor has a story height of 3.25 m. The front façade walls are made of rubble stone with thickness of 0.7 m, without reduction of thickness in the height of the building. However, there is a part between the windows in each floor where the thickness of the front façade wall is only about 0.35 m. The side (gable) walls are made with concrete blocks with thickness of 0.2 m. The interior walls are made by solid brick masonry with the thickness of 0.2 m, whereas the partition walls were 0.1 m of thickness and made with the hollow brick. The back salient shape on the back of the "Placa" building is made of RC beams and columns filled with brick masonry walls and RC slabs. In addition, there are RC lintel beams on top of the windows on the front façade wall. The floor of the building consists of different materials. At the front, the building has a wooden floor, so it has a flexible behavior, whereas on the back of the building, supported by the reinforced concrete frame, there is a concrete slab, thus providing plan behavior more rigid than in the wood floors areas. According with the Descriptive Memory (1939), the concrete slabs have 0.1 m thickness and has reinforcement in both directions.

As already mentioned, the number of experimental campaigns addressed to the assessment of these types of buildings in Lisbon is very scarce (Proença and Gago, 2011). Most of the studies are related to the test of masonry specimens built to reproduce existing walls, as the case of Milošević et al. (2013) and Moreira et al. (2012). It is worth noting that the mechanical characterization of material conducts to a great dispersion of results. In this study, the mechanical masonry parameters were defined based on the experimental results and on the average values proposed in Italian code for rubble masonry, solid and hollow clay brick masonry (MIT 2009). The values in the brackets were defined by applying a reduction factor to elastic values equal to 0.75. This reduction was based on the results of parametric nonlinear finite element method analyses described in Cattari (2007). Values related to stiffness properties (E and G) of masonry panels are representative of a cracked state. Young modulus for rubble stone masonry was adopted according to the performed experimental campaign (Proença and Gago 2011). Values for all adopted parameters are depicted in Table 1.



(a) (b)

(c) (d)

Figure 6 Building type D: (a) front façade; (b) and (c) three-dimensional view and plan geometry; (d) distribution of the walls: green (front façade walls), red (interior and exterior walls), yellow (gable walls), black (partition walls)

Table 1 Mechanical parameters and loads adopted in the numerical model.

Mechanical and geometrical properties	Thickness [m]	Young Modulus [GPa]	Shear Modulus G [GPa]	Comp. Strength f_m [MPa]	Shear Strength τ_0 [MPa]	Specific Weight γ [kN/m^3]	Gravity loads G (Variable loads Q) [kN/m^2]	
Rubble Stone Masonry	0.70	2.00	0.58 (0.44)	3.2	0.065	21.0	Floors	
							1.3 $^{(1)}$ (2.0)	3.0 $^{(2)}$ (2.0)
Solid Clay Brick Masonry	0.20	1.5 (1.13)	0.5 (0.38)	3.2	0.076	18.0	Staircase	
							1.3 $^{(1)}$ (4.0)	3.0 $^{(2)}$ (4.5)
Hollow Clay Brick Masonry	0.10	1.20 (0.90)	0.40 (0.30)	2.4	0.060	12.0	Roof 1.9 (0.4)	
Concrete Block	0.20	2.00	0.74 (0.56)	3.7	0.210	14.0	Balcony 3.0 (2.0)	
Reinforced Concrete	Concrete class: C16/20 Steel Class: A235							

$^{(1)}$ Wood Floor; $^{(2)}$ Concrete Floor

THE EQUIVALENT FRAME MODEL

The modeling of building addressed to assess the seismic global response of the “Placa” building was implemented by means of the equivalent frame method based on the solutions adopted in Tremuri software (Lagomarsino et al. 2013). The numerical models are represented in Fig. 7. This approach starts from the main idea (supported by the earthquake damage survey) that the in-plane response of each masonry walls with openings may be discretized by a set of panels: (i) piers, vertical elements, supporting both dead and seismic loads; (ii) spandrels, horizontal elements (between two vertically-aligned openings) coupling piers in the case of seismic loads; and (iii) rigid nodes, undamaged masonry portions confined between piers and spandrels.

According to this equivalent frame idealization of masonry walls, the insertion of RC structural elements appears quite natural. Fig. 7 shows the idealization of a mixed masonry-RC structure according to this approach. In particular RC nonlinear elements (aimed at modelling RC columns, beams and walls) have been implemented in the program (Cattari 2007).

Since the model herein developed focuses only to the global building response (which is assumed to be governed only by the in-plane behavior of walls), the local flexural behavior of floors and the out-of-plane walls' response are not explicitly computed. Despite this assumption, it has to be highlighted that "local" out-of-plane mechanisms may be verified separately through suitable analytical methods if no strengthen measures of the connections were adopted.

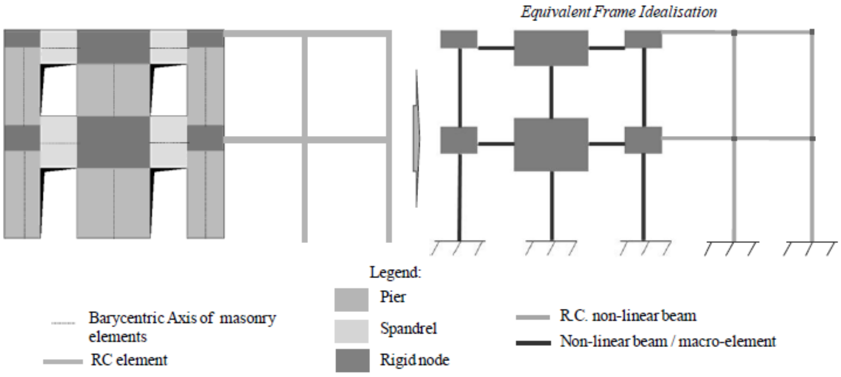


Figure 7 Example of mixed RC-masonry structure with the corresponding equivalent frame idealization (Cattari and Lagomarsino 2013)

The in-plane behavior of masonry piers and spandrels, as well as, the RC elements were modeled by non-linear beams (Cattari and Lagomarsino 2013). Floors were modeled as anisotropic membrane finite elements (Lagomarsino et al. 2013). The timber floors were defined as an equivalent membrane with 0.02 m thickness and characterized by $E_{1,eq}$=20.6 GPa (perpendicular to the façade walls), $E_{2,eq}$=8.0 GPa (in orthogonal direction) and G_{eq} = 0.4GPa intended to simulate flexible behavior. For the RC slabs on the back of the building, the following properties were considered: 29 GPa for E_{eq} equal in both directions and 12 GPa for G_{eq} .

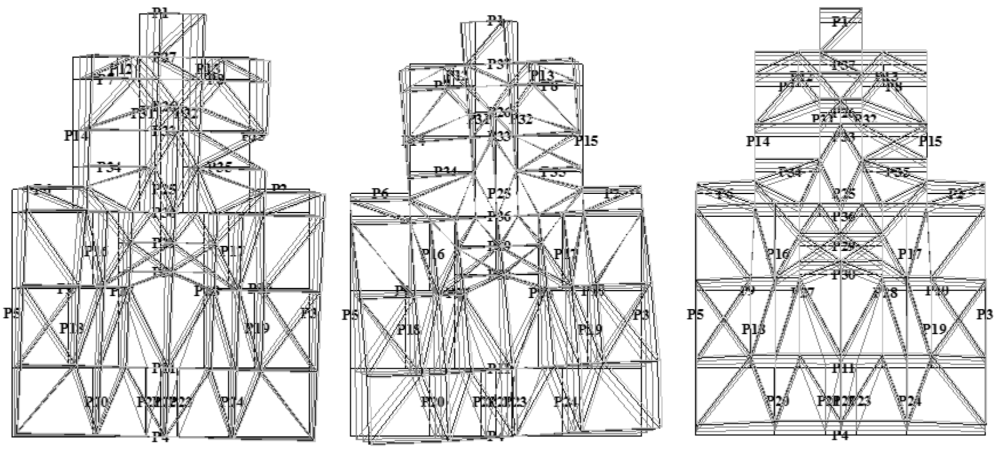
In order to obtain the dynamic properties of the buildings, modal analyses were also performed. In this study, frequencies *f* obtained from the modal analyses were compared with the results, which were obtained by experimental tests (Table 2) (X direction is parallel to the façade wall).

Table 2 Comparison between the frequencies of the model (isolated) and experimental test

<table>
<tr><th>Description</th><th colspan="2">f_x [Hz]</th><th>f_Y [Hz]</th></tr>
<tr><td>Experimental</td><td colspan="2">4.45</td><td>5.10</td></tr>
<tr><td rowspan="2">* G_{eq} = 0. 4GPa</td><td>RC area</td><td>3.01</td><td rowspan="2">4.17</td></tr>
<tr><td>Timber area</td><td>4.12</td></tr>
<tr><td rowspan="2">* * G_{eq} = 0. 4GPa</td><td>RC area</td><td>3.37</td><td rowspan="2">4.67</td></tr>
<tr><td>Timber area</td><td>4.48</td></tr>
<tr><td rowspan="2">* G_{eq} = 0. 04GPa</td><td>RC area</td><td>2.92</td><td rowspan="2">3.98</td></tr>
<tr><td>Timber area</td><td>3.44</td></tr>
<tr><td rowspan="2">* * G_{eq} = 0. 04GPa</td><td>RC area</td><td>3.26</td><td rowspan="2">4.48</td></tr>
<tr><td>Timber area</td><td>3.79</td></tr>
</table>

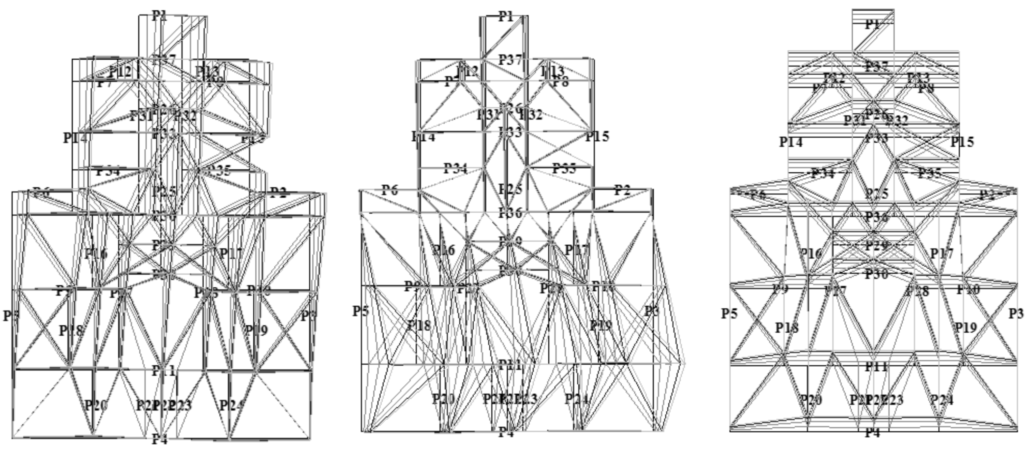
* Cracked state; * * Uncracked state

Furthermore, in order to analyze some uncertainty which can affect the response of the building, some parametric modal analyses were performed and presented. Namely, two different values of shear modulus for timber floor are adopted: G_{eq} = 0.4 GPa and also values ten times smaller, i.e. G_{eq} = 0.04 GPa (representative of the condition in which a very poor connection among structural timber elements is present). Moreover, it is worth noting that for the modal analyses performed different stiffness parameters (E and G) of masonry walls were considered: reduced values taking into account the cracking (with application of a reduction factor of 0.75) and assuming an uncracked state (without the application of any reduction factor). Based on the results obtained, different behavior can be noticed for different modes of vibration in the X direction, comparing the behavior of the back of the building, where RC floors are located, with the behavior of the part of the building with more flexible floors (timber one). According to the analyses of the deformed shape in plan of two first modes for G_{eq} =0.4 GPa (Fig. 8) and G_{eq} =0.04 GPa (Fig.9) (that activate a significant percentage of mass in X direction) is evident that the first one is related mainly to the activation of the area of part with RC slab, while the second one is related to the area with the timber floors. It should be mentioned that in case of experimental test was not possible to notice different behaviors between these two parts of the building.



(a) (b) (c)

Figure 8 Deformed shape of the building (G=0.4 GPa): (a) Mode 1; (b) Mode 2; (c) Mode 3



(a) (b) (c)

Figure 9 Deformed shape of the building (G=0.04 GPa): (a) Mode 1; (b) Mode 2; (c) Mode 3

Based on the results obtained (Table 2), the most adequate model is the one defined for a G_{eq} = 0.4 GPa for the timber floor and for an uncracked state (with elastic properties of modulus E and G). For this model (the correspondent frequencies are marked in bold in Table 2) the values of the analytical

frequencies are closer to the experimental values; nevertheless, in the X direction, there still is a difference between both results.
The obtained differences are connected with the fact that studied building was analyzed as an isolated structure, without influence of adjacent buildings and with different boundary conditions. In case of Y direction, these effects are negligible, thus obtained results are in a good agreement with the experimental one.
Further studies will be carried out in order to get the behavior of the building inside the aggregate, restrained on both or only on one sides with the adjacent buildings. Regarding the previous experience, it is expected to obtain similar results with the experimental one, since that the building will be modelled taking into account all conditions, which did not defined for isolated structure.
Comparing the obtained value of fundamental experimental frequencies with the values of other studies for other types of unreinforced masonry buildings in Lisbon ("Pombalino" and "Gaioleiro" buildings) the presented experimental values obtained for this "Placa" building are really high, since that above mentioned buildings have frequency values around 2 and 3 Hz (Branco, 2007; Oliveira, 2009). However, differences of the structure elements, including the properties of the existent materials and different plan shape of the presented "Placa" building comparing with the "Pombalino" and "Gaioleiro" should be considered. Nevertheless, Oliveira and Navarro (2010) have done in-situ dynamic test on the buildings similar to presented one, where values for frequencies matches well with the obtained experimental results (f_X=4.1 Hz and f_X =4.3 Hz).

NONLINEAR SEISMIC ANALYSES OF "PLACA" BUILDINGS

The seismic performance based assessment is leading to an increasing use of nonlinear static procedures. These concepts are based on the comparison between the displacement capacity of the structure and the displacement demand of a given seismic action. The structure capacity is defined through a force-displacement curve, which describes the overall inelastic response of the structure and provides essential information to idealize its behavior in terms of stiffness, overall strength and ultimate displacement capacity. These curves were obtained by a nonlinear incremental static (pushover) analysis. The analyses were performed in Tremuri Program for each main direction of the building (X and Y in positive direction) considering two load patterns: (i) uniform, proportional to the mass; and (ii) pseudo-triangular, proportional to the product between the mass and height. Fig. 10 shows the pushover curves obtained, the function of the base shear force (V_b) and average displacement of the nodes located at the roof level (d). The analyses were stopped for 20% decay of the maximum base shear force (V_b) (Fig.10) following the criteria proposed in (EC8, NTC 2008). This condition was assumed as reference in order to define the ultimate displacement capacity of the structure.

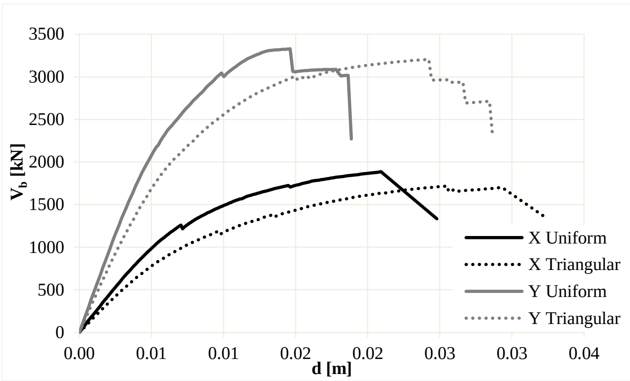


Figure 10 Pushover curves

According to the obtained results, it is clear that the stiffness and strength is much higher in the Y direction than in the X direction, since that the side walls without openings are oriented along the Y axis; thus the beneficial increase of base shear is more pronounced in this direction. In terms of

ductility, higher ductility was obtained on the Y direction. Comparing the results obtained with the two lateral load patterns can be noticed that base shear force values are similar.
The damage distributions obtained for the ultimate displacement and in the some of the exterior and interior walls are depicted in Fig. 11.

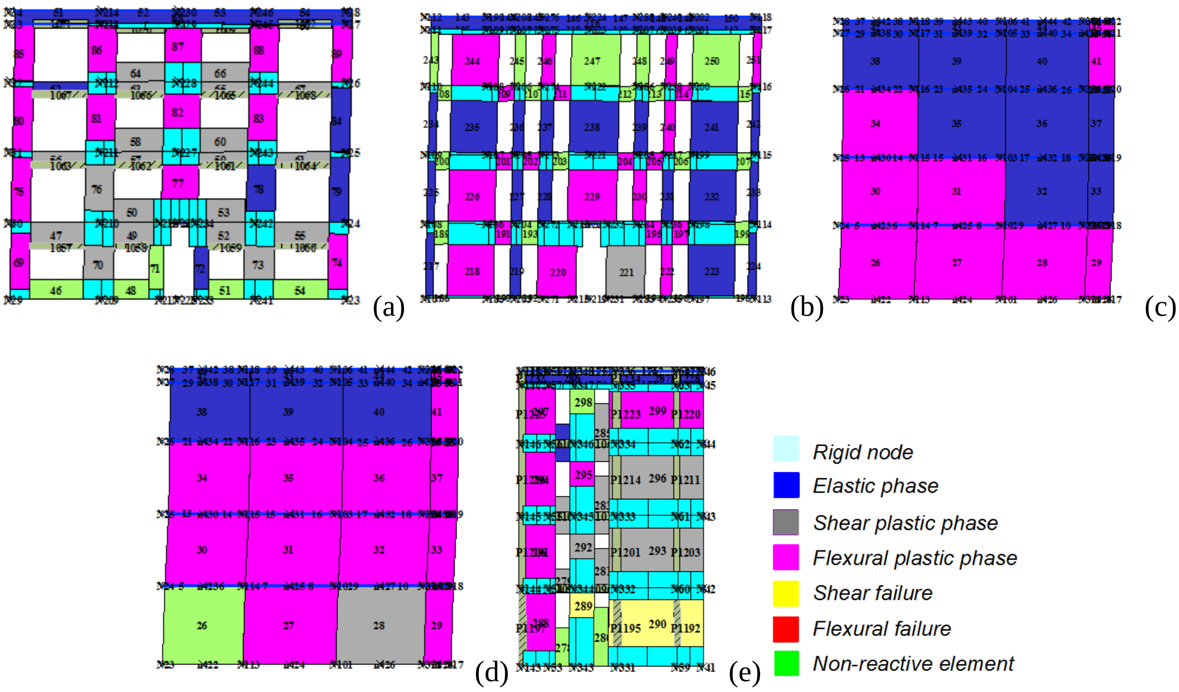


Figure 11 Damage pattern on the building for the ultimate displacement: (a) front façade wall and (b) interior wall on the X direction for the uniform load; (c) and (d) side wall on the Y direction with uniform and pseudo-triangular load, respectively; (e) wall on the back salient shape on the Y direction for the uniform load

On the X direction, the damage pattern on the masonry walls is characterized with shear failure of spandrel beams, due to the presence of the RC lintel beams on the front façade (Fig. 11a). The interior brick walls (Fig. 11b) present disperse damage, mainly due to the flexural behavior, resulting in less damage of façade walls. As shown in Fig. 11(c) and (d), on side walls in the Y direction, piers are characterized, in general, with flexural behavior for both distributions, located mainly in the ground floor in case of the uniform load pattern and spreading on the height of the building in case of the pseudo-triangular load pattern. This can explain the fact that the ultimate displacement obtained with the pseudo-triangular load pattern is higher than the one with the uniform load; therefore, the building was able to explore more the redistribution of the loads and the non-linear behavior of its structural elements with the pseudo-triangular load pattern. Furthermore, in case of the RC frame, existent on the back of the building, damage is concentrated on the ground floor RC columns and on the masonry piers, in both cases due to flexural behavior (Fig. 11(e)).
The displacement performance based assessment (PBA) allows the determination of performance point (or target displacement d^*_{max}), computed from the intersection between the capacity curve and the elastic response spectrum that represents the seismic action in Lisbon. The N2 Method, adopted in both EC8 (CEN, 2004) and the Italian code (NTC, 2008), was considered to assess the seismic performance of this building. An elasto-perfectly plastic force-displacement relationship was assumed to define SDOF capacity curve: the initial stiffness corresponding to the 70% of the maximum base shear reached. The yield force (F_y) was determinate in a way that areas under the SDOF pushover curve and the elasto-perfectly plastic capacity curve are equal. The safety verification consists of checking if the structure can resist the seismic demand defined in (EC8, CEN, 2004) for Lisbon with 475 years return period and 5% critical damping coefficient. The far-field (SA 1.3) and near-field seismic action (SA 2.3) have reference peak ground acceleration respectively equal to 1.5 m/s^2 and 1.7 m/s^2, respectively. The foundation soil is Type C.

Fig. 12 depicts capacity curves on the X and Y direction in the idealized elasto-perfectly plastic relationship. Table 3 shows the properties of the capacity curves for both load patterns: period (T^*) and the ductility (μ, computed by the ratio between the ultimate displacement d_u^* and the yielding displacement d_y^*).

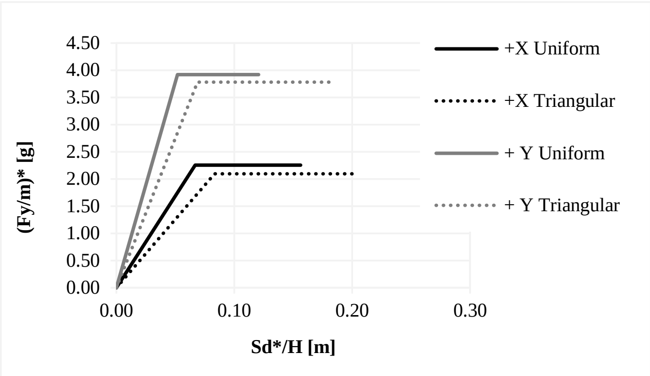


Figure 12 Capacity curves

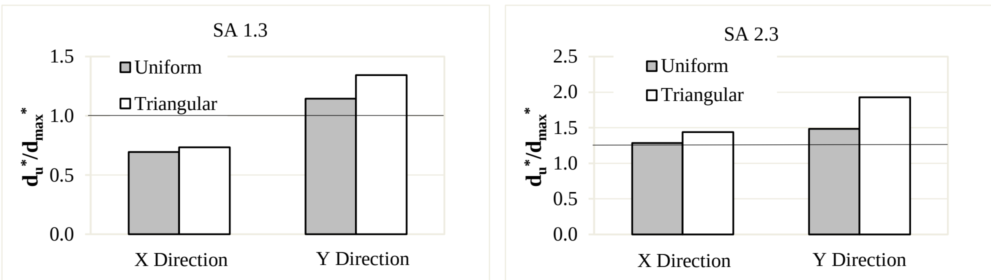
Table 3 Properties of the capacity curves

Capacity Curves	Uniform Load				Pseudo-Triangular Load			
	X Direction		Y Direction		X Direction		Y Direction	
	T^*	μ^*	T^*	μ^*	T^*	μ^*	T^*	μ^*
‘Placa’	0.38	2.34	0.25	2.33	0.44	2.43	0.29	2.67

In terms of period, is noticed, that X direction presents higher equivalent period, which can be explained by the fact that in this direction, structure is more deformable (higher number of openings on the façade wall). Regarding the results obtained for ductility, differences are not so evident.

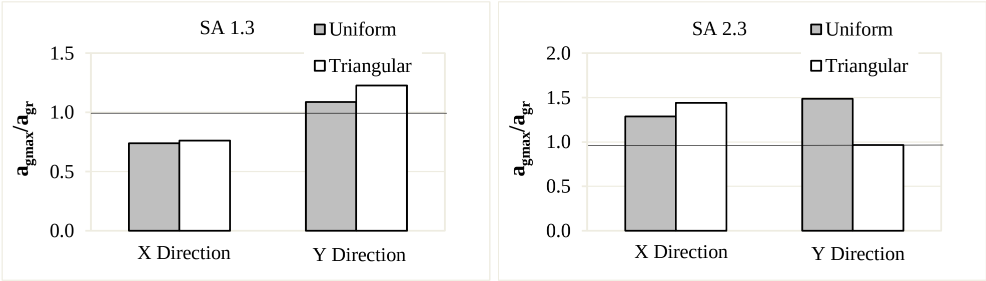
In Fig. 13 the ratio between the ultimate (d_u^*) and the performance displacement ($d_{\max}^*$) is shown for both type of action. According to the PBA, safety is verified if $d_u^* / d_{\max}^*$ >1. As can be noticed, the far-field action (SA 1.3) is the most demanding case. Regarding the obtained results, it can be seen that in case of Y direction no-collapse requirement are satisfied. Since the requirements must be fulfilled for all cases, the building does not verify already mentioned requirements and for that reason needs to be retrofitted.

Fig. 14 shows the ratio between the maximum admissible ground acceleration ($a_{g\max}$) (taking into account q^* < 3) and the reference ground acceleration (a_{gR}), which in case of seismic action 1.3 is equal to 1.5 m/s^2 and for seismic action 2.3 is $1.7\, m/s^2$. If $a_{g\max}/a_{gR}$ > 1, safety is verified. However, as can be seen in Fig. 14 only Y direction satisfied this requested rule. According to the results, it is confirmed that the maximum admissible ground acceleration is much lower than the reference values of the seismic demand corresponding to the far-field action (SA 1.3).



(a) (b)

Figure 13 Ratio between ultimate and performance displacement: (a) far-filed action (SA 1.3); (b) near-field action (SA 2.3)



(a) (b)

Figure 14 Ratio between the maximum admissible ground acceleration and the reference ground acceleration: (a) far-filed action (SA 1.3); (b) near-field action (SA 2.3)

CONCLUSIONS

Due to the limited number of information about seismic vulnerability of mixed masonry-reinforced concrete “Placa” buildings in Lisbon, in the present paper the seismic performance of one building, representative of one typology, has been analyzed through non-linear static analyses and adopting the equivalent frame model. In the current work model focuses only on the global seismic response of the building (which is assumed to be governed only by the in-plane behavior of walls); the local flexural behavior of floors and the out-of-plane walls’ response are not explicitly computed. Despite this assumption, it has to be highlighted that “local” out-of-plane mechanisms may be verified separately through suitable analytical methods. The capacity curves were defined and compared for different load patterns and for both main directions.

Modal analyses were also performed and compared with the values obtained by experimental tests. In order to analyze the influence of the mechanical properties values in the response of the building, parametric modal analyses were performed considering different values of shear modulus for timber floor (G) and different E and G values for all the other materials of the masonry panels, representative of a cracked state (with application of 0.75 factor) and uncracked state (without the application of 0.75 factor). Differences between modal and experimental results are evident, mainly due to the fact that in the current study the building was analyzed as isolated structure.

The “Placa” building (Type D) studied shows that stiffness and strength is higher in the Y than in the X direction. This can be explained by the fact that in X direction, greater area of openings can be found comparing to the X direction.

Regarding the failure mode, it can be concluded that the evolution of the damage is, in general, characterized by the failure of spandrel beams by flexure (apart when coupled to RC lintel beams that instead promote the occurrence of a shear failure type), followed by the collapse of piers mainly by flexure in the lower stories of the building.

In terms of safety verification, can be seen that structure does not satisfy the no-collapse requirement for far-field action, what leads to the conclusion that some appropriate techniques of retrofitting has to be done. As already mentioned, this building was studied as an isolated structure, what is not case in reality, where the building located in the aggregates restrained on both sides by the adjacent buildings.

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